

Memorandum

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Division of Structure Design
Design Office 59-232

Attention Mr. Gary Blakesley

Date: January 26, 2001

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11-0301U1

Southbound 5 Truck Connector
Bridge No. 57-1028F

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Division of Structural Foundations - MS 5
Office of Structure Foundations

Subject: Revised Foundation Recommendations

Introduction

The proposed Southbound 5 Truck Connector (Br. No. 57-1028F) partially completes earlier planned Route 5/56 Interchange improvements for the San Diego/Del Mar area. The proposed Southbound 5 Truck Connector will connect southbound Rte. 5 truck traffic to the existing Route 56/5 Separation (Br. No. 57-0989F) and should help ease congestion in the area. A Request for Final Foundation Recommendations (dated October 22, 1998) for the subject bridge was submitted to the Office of Structure Foundations (OSF) by Mr. Ramin Rashedi (Bridge Engineer). Site specific ARS, liquefaction potential, and methods of liquefaction mitigation were requested in the above memorandum. A list of preliminary column/pile loads and shaft/pile diameters were provided to OSF by Mr. Ramin Rashedi (dated December 18, 1998). As the 5/805 and 5/56 project has progressed, further revisions of the above pile load and shaft/pile diameter list was sent to OSF including Revision 1 (dated February 24, 1999), Revision 2 (dated April 9, 1999), and Revision 3 (dated May 11 and 26, 1999). Due to potential liquefaction at the bridge site, cast-in-steel-shell (CISS) piles, 355 mm (14 in) diameter, in combination with stone columns were originally proposed for bridge support (refer to Revisions 1 through 3 above). Mr. Arturo Jacobo (District 11 Project Engineer) mentioned (May 17, 1999) that proposed stone column construction would conflict with utilities (such as fiber optics), some of which could not be relocated. District 11 Project Development, the Division of Structures Design (DSD), and the OSF agreed that large diameter pile shafts without additional stone columns, would resolve the potential conflicts with utilities and provide an acceptable bridge foundation at the site. The pile shaft alternative without stone columns and final bent pile diameter are noted in Revision 3 (Rashedi, May 26, 1999). Abutment pile diameter and axial service load were provided by Mr. Ramin Rashedi (February 1, 2000) who also requested P-Y curves or COM624 soil profile information at Abutment 12. Mr. Gary Blakesley (Bridge Engineer) provided final bottom of footing/pile cutoff elevations for the proposed bridge (Caltrans facsimile copy, dated February 17, 2000). P-Y curves were also requested by Mr. Earl Seaberg, (Senior Bridge Engineer, DSD) on February 24, 1999. In preliminary evaluations of the As Built Log of Test Borings (LOTBs) from an adjacent structure (Carmel Valley Creek Bridge-Replace, Br. No. 57-0590) performed by the Office of Geotechnical Earthquake Engineering (OGEE) (Perez-Cobo and Abghari, February 10, 1999), potentially liquefiable soils are estimated as approximately 7.6 to 9.1 m (25 to 30 ft) thick.

An additional Request for Revised Foundation Recommendations for Bent Supports 7 through 11 was received January 9, 2001 (Blakesley). Within the above memorandum Structure Design decreased bent pile nominal loads (except Bent 6 where pile nominal resistance remained unchanged) and increased Abutment 12 pile design loading. Pile diameters and finished grades at abutments and bents remained unchanged.

Subsurface information was obtained by OSF drilling and sampling eight 89 to 94 mm diameter mud rotary borings which also involved extensive coring. Results from the field studies will be shown on the LOTBs. In addition to the recent field work, the LOTBs for the adjacent Carmel Valley Creek Bridge-Replace (Br. No. 57-0590) and the Rte. 56/5 Separation (Br. No. 57-0989F, expenditure authorization No. 11-030111, approved March 8, 1993), and the Carmel Valley Road Undercrossing (Br. No. 57-486R/L, Contract No. 11-022484, approved April 1, 1963), contained additional site and subsurface information and will be included within the new contract plans.

Site Description

During OSF's recent foundation investigation, sediments at the site consist of a preexisting embankment fill slope from adjacent Rte. 5 in the area of proposed Abutment 12. Embankment fill material ranges between approximately 5.18 to 7.32 m (17 to 24 ft) thick in the proposed Abutment 12 area (north). At the proposed bent areas, artificial fill ranges from approximately 2.44 to 5.18 m (8 to 17 ft) thick. Underlying Holocene estuary deposits, Holocene and possible older Quaternary alluvium (undifferentiated), and probable Pleistocene Bay Point Formation remnants (undifferentiated), range from approximately 13.11 to 31.70 m (43 to 104 ft) thick and generally thin to the north. The undulatory top surface of the underlying Eocene Delmar Formation was encountered from elevations -6.64 m (-21.8 ft) in the proposed Abutment 12 area (north) to -29.50 m (-96.8 ft) at the proposed Bent 6 area (south).

Approach embankment fill material consists dominantly of dense to medium dense and minor loose, sand with intermittent scattered gravel. At the proposed bents, artificial fill generally consists of loose to very dense /stiff, sand with intermittent scattered gravel interlayered with gravel and minor lean clay with sand. Artificial fill also contains minor shell fragments, roots, and wood fragments. Native material, mapped as Holocene estuary deposits, Holocene and possible older Quaternary alluvium and slope wash (undifferentiated), and probable Pleistocene Bay Point Formation remnants (undifferentiated), according to Power and others (1982), Kennedy (1975), and OSF's recent drilling program, can be roughly divided into two units. The upper sediments range from approximately 0 to 16.76 m (0 to 55 ft) thick and consists of dominantly loose to medium dense/soft to very stiff, sand, silty sand, and sandy silt with intermittent scattered gravel interbedded with sandy lean clay, clayey sand, minor elastic silt, and clay. Whole and fragmented bivalves, snails, and roots were found. Some organic layers were odoriferous within this unit. The underlying sediments range from 5.18 to 16.46 m (17 to 54 ft) thick and are found below elevations ranging from +6.40 to -13.41 m (+21 to -44 ft). Underlying sediments consist of generally medium dense to very dense/stiff to hard sand with intermittent gravel, gravel/cobble (hard metavolcanic rock fragments up to 150 mm diameter) lenses with sand and/or clay matrix, silty sand, silt, and clayey sand interbedded with sandy clay, clay with rare gravel, and minor elastic silt. Sporadic intensely weathered zones, blocky texture, roots, and iron staining of soils was present within the lower sediments which probably represent older alluvium or Pleistocene Bay Point Formation deposits. Much of the upper sediments are considered potentially liquefiable and are being investigated by the OGEE for potential mitigation measures or adequacy of proposed mitigation measures. As mentioned earlier, final p-y (lateral resistance) curves are also being developed for use at proposed bridge support locations. The underlying Eocene Delmar Formation generally consists of interbedded very soft to moderately hard, mudstone, claystone, siltstone, and sandstone. The formation is intensely to slightly weathered, generally unfractured to slightly fractured, uncemented to moderately well cemented, often thinly bedded, and contains occasional concentrations of pelecypod debris. The interbedded sandstones can be either uncemented and soil-like (very dense sand) or calcite-cemented and rock-like (increasingly rock-like with increasing depth). The very soft to soft upper formation mudstones, claystones, sandstones, and siltstones of the Delmar Formation [approximately 7.01 to 12.19 m (23 to 40 ft) thick], were considered to possess weak to very weak

rock unconfined compressive strengths ranging from 552 to 1068 kPa (80 to 155 psi). Below this upper zone, generally soft to moderately hard mudstone, claystone, sandstone, and siltstone (weak to fairly strong rock) show unconfined compressive strengths ranging from at least 1379 to 2758 kPa (200 to 400 psi) and higher, increasing with depth. The deepest boring for the bridge, Boring 99-6 (near proposed Bent 11), was 60.81 m (199.5 ft) below the surface [elevation -53.37 m (-175.1ft)]. Downhole P-S logging (compression and shear wave) showed that the better quality intermediate (interbedded soil-like and rock-like material) to rock-like formational material had shear wave velocities ranging from 548 to 945 meters per second (1800 to 3100 fps) which appeared to correlate with unconfined compressive strengths of at least 1379 kPa (200 psi) and generally higher than 1723 kPa (250 psi), up to 3447 kPa (500 psi). The higher shear wave velocities in the above pseudo-rock-like and rock-like material were found in the Delmar Formation below approximate elevations ranging from -23.16 to -33.83 m (-76 to -111 ft). The LOTBs should be reviewed for more specific details.

Ground Water

Static ground water was last measured on January 11, 2000, within Boring 99-1 (near proposed Bent 6) at elevation +3.81 m (+12.5 ft). The ground water level fluctuated approximately 0.09 m (0.3 ft) during OSF's recent investigation.

The As Built LOTBs for the Carmel Valley Road Undercrossing (Br. No. 57-0486R/L) shows ground water was encountered from elevations +1.40 to +0.67 m (+4.6 to +2.2 ft) based on the City of San Diego datum, which requires a +2.48 m (+8.15 ft) add (Schuh, Caltrans Memorandum, February 14 and March 7, 2000) to adjust to the current metric elevations (NAVD 88) upon which the recent plans and boring program are based. The adjusted to metric As Built elevations would then show ground water was encountered at elevations +3.88 to +3.15 m (+12.8 to +10.4 ft) for the earlier foundation investigation, with measurements taken during April 1962. The LOTB for the more recently completed Rte. 56/5 Separation (Br. No. 57-0989F) reveals ground water was encountered within Boring R-36 (near existing Bent 5 for the Southbound 5 Truck Connector) at elevation +3.08 m (+10.1 ft), measured May 17, 1991. Correcting English (NGVD29) to metric (NAVD88) elevations would show ground water encountered at elevation +3.70 m (+12.1 ft). The correct add from English (NGVD29) to metric (NAVD88) would be 0.619 m (2.03 ft) according to Schuh (February 14, 2000).

Seismicity

See the memorandum (dated February 10, 1999) concerning Preliminary Seismic Design Recommendations sent to Mr. Earl Seaberg (Senior Bridge Engineer) from Mr. Angel Perez-Cobo and Dr. Abbas Abghari OGEE section. Final Seismic Design Recommendations and Lateral Resistance, p-y Curves, will be submitted by the OGEE.

As mentioned above (Perez-Cobo and Abghari, February 10, 1999), the proposed "structure is located approximately 5 km from the Newport-Inglewood-Rose Canyon fault which has a maximum credible earthquake moment magnitude of $M=7.0$ and based on the Caltrans California Seismic Hazard Map (Mualchin, 1995), this structure will be located within the peak horizontal bedrock acceleration zone of 0.5 g."

As mentioned above pseudo-rock-like material [Vs ranging from 548 to 945 meters per second (1800 to 3100 fps)] occurs below approximate elevations ranging from -23.16 to -33.83 m (-76 to -111 ft).

Liquefaction

Liquefaction potential is considered moderate to high. Holocene estuary and alluvial deposits and possible older Quaternary alluvium (undifferentiated) at the site are dominantly composed of loose to medium dense/soft to very stiff, sand, silty sand, and sandy silt with intermittent scattered gravel interbedded with sandy lean clay, clayey sand, minor elastic silt, and clay. The above sediments are up to 16.76 m (55 ft) thick. Ground water is also rather shallow [measured within the recent investigation at 1.74 m (5.7 ft) below the surface at Bent 6 (Boring 99-1)]. Preliminary analysis (Perez-Cobo and Abghari, February 10, 1999) estimates that the top 7.62 to 9.14 m (25 to 30 ft) of soils are considered potentially liquefiable. As mentioned above, final liquefaction potential is being determined by the OGEE.

Foundation Recommendations

The following recommendations are based on the Southbound 5 Truck Connector (Br. No. 57-1028F) General Plan (revised May 1, 1999), Foundation Plans (2 sheets, checked by S. Wang, January 22, 1999), the above mentioned memorandums and personal communications from Messrs Ramin Rashedi (Caltrans facsimile copy dated May 26, 1999 and a memorandum supplying abutment pile diameters and service loads, February 1, 2000), and Gary Blakesley (Caltrans facsimile copies with final bottom of footing elevations, dated February 17 and March 24, 2000). Requested revisions (Blakesley, January 9, 2001) are discussed within the Introduction and reflected within the Pile Data Table below.

Fills can be placed in accordance with Section 19-6 of the Standard Specifications. End dumping is not permitted. At the Abutment 12 area, additional fill is estimated at 2.74 to 5.18 m (9 to 17 ft) height. Existing embankment for the Carmel Valley Road Undercrossing (Br. No. 57-0486R/L), adjacent to the Abutment 12 area, has been in place since 1964 so settlement will be reduced somewhat from the calculated maximum settlement of 203 mm (8 in). An alternate settlement calculation method (Hough's Method) shows a maximum settlement of 51 mm (2 in). The settlement period is estimated at approximately 30 days, however the actual settlement period will be determined by the project engineer on the basis of settlement data in the field.

Structure approach slab type N (9S) will be incorporated within the new bridge.

Plumb, 900 mm (3 ft) diameter, Cast-in-Drilled-Hole (CIDH) Piles can be used to support bridge Abutment 12. Plumb, 3.0 m (10 ft) diameter drilled shafts will be used at the bent supports as shown below. Cast-in-Drilled-Hole Pile capacities were calculated using the Federal Highway Administration's Drilled Shaft Manual (Pub. No. FHWA-HI-88-042) published July 1988. Design loading (Working Stress Design) at the abutment and nominal resistance (Load Factor Design) at the bents were supplied by the Division of Structure Design (Rashedi, Caltrans Memorandum dated February 1, 2000 and facsimile copy and personal commun., Revision 3, May 26, 1999). Mr. Gary Blakesley (February 17 and March 24, 2000, and January 9, 2001) provided final bottom of footing/pile cutoff elevations and revised bent pile nominal resistances and abutment pile design loading.

Southbound 5 Truck Connector, Br. No. 57-1028F:

Support Location/ Type & Diameter	Design Loading			Nominal Resistance		Bottom of Pile Footings/Cutoff Elevation m (ft)	Begin Pile Bearing Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
	Compression kN (tons)	Tension KN (tons)	Lateral KN (tons)	Compression kN (tons)	Tension KN (tons)				
Bent 6/ CIDH 3.0 m (10 ft)				31,600 (3552.0)		+5.00 (+16.4)	-13.41 (-44.0)	-41.45 (1) (-136.0)(1)	-41.45 (-136.0)
Bent 7/ CIDH 3.0 m (10 ft)				26,000 (2922.5)		+7.00 (+23.0)	-11.89 (-39.0)	-38.40 (1) (-126.0)(1)	-38.40 (-126.0)
Bent 8/ CIDH 3.0 m (10 ft)				31,000 (3484.6)		+7.00 (+23.0)	-13.41 (-44.0)	-34.44 (1) (-113.0)(1)	-34.44 (-113.0)
Bent 9/ CIDH 3.0 m (10 ft)				28,000 (3147.3)		+7.00 (+23.0)	-10.06 (-33.0)	-34.14 (1) (-112.0)(1)	-34.14 (-112.0)
Bent 10/ CIDH 3.0 m (10 ft)				26,000 (2922.5)		+6.70 (+22.0)	-3.35 (-11.0)	-32.61 (1) (-107.0)(1)	-32.61 (-107.0)
Bent 11/ CIDH 3.0 m (10 ft)				21,000 (2360.5)		+6.70 (+22.0)	+1.83 (+6.0)	-23.77 (1) (-78.0) (1)	-23.77 (-78.0)
Abut 12/ CIDH 900 mm (3 ft)	1350 (151.7)	0		2700 (303.5)		+13.00 (+42.7)	+6.40 (+21.0)	-7.92 (1) (-26.0) (1)	-7.92 (-26.0)

Notes: Design tip elevation is controlled by the following demands: 1 Compression; 2 Tension; 3 Lateral Loads

If pile tip elevation is controlled by lateral demands, the designer is responsible to present correct foundation data, governed by lateral control, on the foundation plans.

Axial compression values noted in the table above are based on skin friction only within unliquefiable soil and underlying rock. End bearing was not considered due to working below the water table and the possibility that cleaning out the bottom of pile borings effectively may be rather difficult at depth and may make it difficult to realize substantial end bearing using Caltrans standard pile vertical deflection criteria of 13 mm (0.5 in).

Constructability

Moderate to possible heavy caving and hard slow drilling through hard metavolcanic gravel/cobble zones (cobbles up to 150 mm diameter) and bedrock is anticipated during installation of CIDH piles. Temporary casing may be considered by the contractor to aid in CIDH pile installation.

The Caliper log within Boring 99-2 (proposed Bent 7) which was an uncased hole, shows that caving happens readily within shallow loose to medium dense often saturated artificial fill, alluvium, and possible estuary deposits. At least moderate caving occurred between elevations +3.66 to +2.13 m (+12 to +7 ft), and near elevation -10.36 m (-34 ft). The above elevations represent caving near the base of artificial fill and within loose to medium dense, granular alluvial material. The Caliper log within Boring 99-6 (proposed Bent 11) which was also an uncased hole, shows that caving or washouts occur within alluvial sand and gravel between elevations +4.4 to -4.05 m (+14.6 to -13.3 ft), over a zone approximately 8.53 m (28 ft) thick.

Ground water should be anticipated at shallow depths. Static ground water was measured at elevation +3.81 m (+12.5 ft) within Boring 99-1 (drilled near proposed Bent 6). Water will probably be encountered during CIDH pile construction. The bottom of all excavations should be cleaned of loose debris before placing concrete.

Clay mineralogy within formational material appears sensitive to the introduction of fresh water, which could cause swelling of clays and slicking of borehole walls, resulting in reduced pile/soil skin friction capacity. OSF feels that a mud/polymer expert should be consulted and be available to the contractor to advise on proper drilling fluid/slurry chemistry in order to prevent clay swelling.

Corrosiveness

Laboratory tests of composite soil samples, taken within Boring 99-2 (proposed Bent 7) and 99-4 (proposed Bent 8), indicate that native estuarine and alluvial deposits are sporadically corrosive. Corrosion tests on native material show pH ranges from 5.2 to 8.6, minimum resistivity ranges from 470 to 1700 ohm-cm, sulfate content ranges from 25 to 1620 ppm, chloride content is less than 25 ppm, and years to perforation of 18 gauge galvanized steel culvert ranges from 5 to 28 years. Soil samples were taken along the estimated pile lengths.

As mentioned in the Foundation Recommendations for the nearby Carmel Valley Creek Bridge – Replace (Br. No. 57-0590) by Mr. Jeff Knott (Office of Engineering Geology – South, September 18, 1992), “considering the depositional environment, concrete below ground should be resistant to sulfates and organics.”

Caltrans Corrosion Technology Branch has provided detailed corrosion review and corrosion recommendations for the Southbound 5 Truck Connector, Br. No. 57-1028F (Tolin, July 11, 2000). The above memorandum should be consulted for corrosion mitigation measures regarding CIDH piles, pile caps, walls, and footings.

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If you have any questions, please call Joe Pratt at (562) 864-5740 or Richard Fox at (916) 227-7085.

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